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Structural development and testing of a prototype house using timber and straw bales

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Abstract

One approach to reducing embodied carbon dioxide of buildings is the increased use of plant-based construction materials such as prefabricated straw bale panels. This paper presents findings from the development and structural testing of an innovative load-bearing prefabricated straw bale building. Work on panel development is summarised ahead of presenting two numerical computer-based models that support the building design. The computer models are validated using data from a full-scale simulated static wind load test on a two-storey building. The prefabricated straw bale structural system is shown to be suitable for two- and three-storey domestic structures in a range of locations.

1 Introduction

The development of more operationally energy-efficient buildings is often coupled with a significant increase in the embodied energy and carbon of the building fabric. By 2020 it is predicted that, if current trends continue, the embodied carbon of new buildings will far exceed that from operational emissions over a 60-year design life (Sturgis and Roberts, 2010). In part this is attributed to increased use of high embodied energy insulation materials. It is becoming increasingly clear that the delivery of low carbon buildings also requires the use of low carbon materials. One approach to reducing embodied carbon is the greater use of plant-based construction materials because photosynthetic materials use atmospheric carbon dioxide during their growth. Wheat straw typically sequesters around 1.35kg of carbon dioxide per kg of baled material at 10% moisture content (Sodagar et al., 2011). This carbon dioxide remains effectively stored within the material in a building. Contemporary straw bale construction thus offers the potential for increasing the use of low-impact plant-based materials in modern buildings. However, with the exception of timber and bamboo, the modern use of plant-based materials in construction is limited.

Historically, straw has been used within buildings for thousands of years in applications such as thatched roofs, lath and as

reinforcement for earthen materials. However, it was not until the late 1800s in Nebraska that straw bales were first utilised as load-bearing walls. The use of bales in this way followed the development of mechanical baling machines in response to a shortage of other vernacular building materials. In these early applications the straw bales were used as large lightweight masonry blocks laid in courses and subsequently rendered with a clay plaster to form load-bearing walls (King, 2006). Over the past 20 years straw bale building has been experiencing a revival, with successful projects in California numbering a few thousand. Contemporary straw bale buildings in the UK are now estimated to number well over one hundred.

Further to the low embodied carbon of straw bales, other benefits to use in buildings include:

- high levels of thermal insulation; $\lambda = 0.052\text{--}0.080 \text{ W}/(\text{mK})$ (Lawrence et al., 2013)
- low material cost
- value-added use of a widely available and sustainable

- co-product of food production
- robust fire resistance (Wall et al., 2012)
- provision of a vapour permeable external envelope.

There are, however, some notable limitations associated with using straw bales in construction projects. The strength and stiffness of load-bearing straw bale walls typically limit the height of this building form to two storeys and the reliance on a variable product supply chain means that a consistent bale product is difficult to guarantee. Nonetheless, when straw is used as an insulation infill within a load-bearing frame these issues can be controlled and much more flexibility is afforded to engineers and architects. Goodhew et al. (2010) provide a wider discussion of the benefits and challenges of using straw bales in construction.

The traditional in situ use of straw bales, either for load-bearing or insulation infill, presents significant challenges for wider acceptance in the UK. Straw needs to be kept sufficiently dry throughout its life, which often necessitates the use of temporary shelter during site works. As an agricultural co-product straw bales lack the consistency generally expected from other building products; bale lengths can vary by as much as 100 mm. In recognition of these concerns, and to make straw bale construction more acceptable to wider industry, a number of varying prefabricated panel solutions have been developed worldwide. One of the oldest and most successful of these is the ModCell panel system (IPO, 2003).

As a co-product of the farming industry, straw is a natural, renewable and biodegradable material that requires little processing other than baling. Current usage of straw bales in construction has a negligible impact on the existing supply chain. A National Non-Food Crops Centre (NNFCC) report in 2009 outlined that approximately 3 Mt of straw are returned to the land (soil conditioning) every year. This has been estimated as sufficient to build around 1.5 million houses (NNFCC, 2009). Demonstrably there are adequate supplies of straw available for a significant number of buildings to utilise this material. Nonetheless, it is acknowledged that diverting much larger quantities than currently used into construction may require greater use of fertilisers (in place of soil conditioning) and potentially impact on other current uses, such as animal bedding and mushroom cultivation. The use of straw as biomass for electricity generation and as a feedstock for bio-fuel production has also begun in recent years and is expected to increase with government support.

This paper begins by outlining the development, construction and structural performance testing of individual prefabricated straw bale panels, followed by the development, testing and modelling of a two-storey prototype house. The aim of this paper is to enhance the structural understanding of prefabricated straw bale construction for wider use in the UK and beyond.

2 Development and manufacture of prefabricated straw bale panels

The use of prefabricated building elements, including wall panels, removes many existent barriers to the wider acceptance of straw bales in modern low carbon construction. Prefabricated panels completely remove the need to work with straw on site, providing protection to the straw from inclement weather once the panels leave the manufacturing facility. Timber framed panels also provide a higher quality product of regular and consistent dimensions more suited to the needs of modern construction than the more irregular bales. In recognition of the benefits of this approach, there are a growing number of prefabricated straw bale panel systems in use in Europe, North America and Australia.

The structural frames are most commonly timber, with typically either box or solid engineered timber sections most prevalent (Figure 1). In these uses the straw is primarily used as low carbon infill insulation. Panels are typically finished with a lime-based render or dry lined with a timber sheathing board. The panels are used to form both load-bearing and non-load-bearing walls and typically only form the external envelope of a building. In loadbearing applications the timber frame is designed to carry vertical floor and roof loads and in non-load-bearing applications the panels are used as a cladding fixed back to a more conventional structural frame.

ModCell was one of the earliest prefabricated straw bale panel systems, first used in the University of the West of England School of Architecture and Planning in 2001. Originally developed as a low embodied carbon energy efficient cladding solution, recent research and development has enabled ModCell panels to also be used in load-bearing applications.

ModCell panels are typically formed using a softwood glulam timber frame (C24 grade) and measure 3.20 m by 2.6–2.9 m high and 0.48–0.49 m thick. The dimensions of the timber frame and connections vary depending on application, but current loadbearing frames are formed from 100 mm thick

glulam vertical members and 160 mm thick header beams. The header beam sections are sized based on the applied load and can vary between 100 mm and 240 mm (Figure 1). The base plates are formed from 100 mm thick glulam. The timber panel members are connected at the four corners using 8 mm diameter self-tapping screws with a minimum embedment length of 100mm in the adjoining member.



Figure 1. Glulam frame of a ModCell panel

Once the glulam frame has been formed, the dry and compressed straw bales are laid in a running bond within the panel. In order to provide robustness to the panel, 20–25 mm diameter timber stakes are used to stake bale courses together. The same stakes are used to connect the glulam frame to the staked straw bale infill. Stainless steel (12 mm diameter) threaded bars are used to control top plate deflection and to brace the panel corners. Once the panels have been filled with straw they are finished with a 30 – 35 mm thick formulated lime render, which is spray applied directly onto the straw in three coats. The primary function of the render is to protect the straw from exposure to moisture, insect and rodent attack and for additional fire protection. However, the render coatings also provide a substrate for lightweight fixings and, as with non-panelised straw bale walling, the render enhances structural capacity. The 28-d flexural strength and compressive strength of the formulated lime render has been measured at 1.33 N/mm² and 3.14 N/mm² respectively (Gross, 2009). The render achieves 50% of its final strength after only 7 d, and achieves its full strength after 14 d.

The modular-sized panels facilitate design and construction, but can incorporate varying amounts of straw bale insulation and openings (glazing and doors), all incorporated during panel prefabrication. To simplify detailing, the openings are typically full panel height. ModCell panels are described here by the number of lengthwise bales used to infill the glulam frame. A ‘three-bale’ panel is completely infilled with straw bales; requiring three standard bales to make up the panel. The ‘two-bale’ (Figure 2) and ‘one-bale’ panels are similarly made with the corresponding quantity of straw bales together with the openings.



Figure 2. Typical two-bale panel

3 Development and experimental validation of structural performance

Since their initial application for cladding, starting with the University of West of England building in 2001, the ModCell panels have undergone further development to improve their strength and stiffness when subject to vertical and, in particular, lateral loading. These developments are described in detail by Gross (2009) and Lawrence et al. (2009). The most significant changes can be summarised as

- increase in thickness of timber panel members from 80– 100 mm to 100–160 mm

- enhancement in strength and stiffness of corner connections through the use of 8 mm diameter 260 – 300 mm long, washer-head, structural screws
- full-panel cross-bracing using threaded stainless bar replaced with shorter corner bracing elements; avoiding overlap of bracing bars
- reduction in thickness of render from 40 – 45 mm to 30 – 35 mm thickness owing to use of corner bracing
- experimental validation of structural performance under vertical loading, racking loading and out-of-plane lateral loading.

The engineered timber frames are the primary structural elements in the system. The frames are designed to carry all vertical loadings. Uniformly distributed suspended floor and roof loadings are supported by the top header plates. These are designed to carry the distributed floor and roof loads onto the frame’s vertical members within set deflection limits. This ensures that vertical load transfer to the render is controlled to a level that prevents damage to the protective finish of the straw.

When resisting in-plane and out-of-plane lateral wind loads structural resistance is reliant on the development of a composite interaction between the lime-rendered straw infill and the timber frame. The lime-rendered straw bale infill must withstand out-of-plane wind loading without cracking of the brittle protective finish occurring. The rendered bales also contribute to the in-plane racking resistance of the panels in combination with the frame and stainless steel corner bracing (Lawrence et al., 2009).

When subject to out-of-plane loading the rendered straw bale behaviour is seen as analogous to a stressed skin (King, 2006). Testing at the University of Bath under simulated out-of-plane wind loading has demonstrated that a 3.3 m ‘three-bale’ panel can safely resist uniform static equivalent wind pressures above 2 kN/m² without cracking. The wind pressures are designed to be transferred from the infill to the glulam frame through 20–25 mm timber dowel connectors, although some arching action and friction between the straw, lime render and timber frame can also be expected. These dowel connectors are spaced vertically at every bale course (typically 350 mm c/c) and driven approximately 400 mm into the straw and have demonstrated sufficiency in testing. Although larger ModCell panel sizes up to 5.3 m have been proposed, further testing and possible refinement of design may be required. For most applications the standard ModCell 333m panel has more than sufficient out-of-plane wind load resistance.

As well as resisting out-of-plane lateral loading the ModCell structural system also requires the panels to resist in-place (racking) forces. Development of sufficient racking resistance has been the key focus of recent research with a primary aim of increasing stiffness. A number of full-scale racking tests, with varying internal bracing arrangements, have been completed to date (Gross, 2009; Lawrence et al., 2009). A ModCell panel under racking testing is shown in Figure 3.

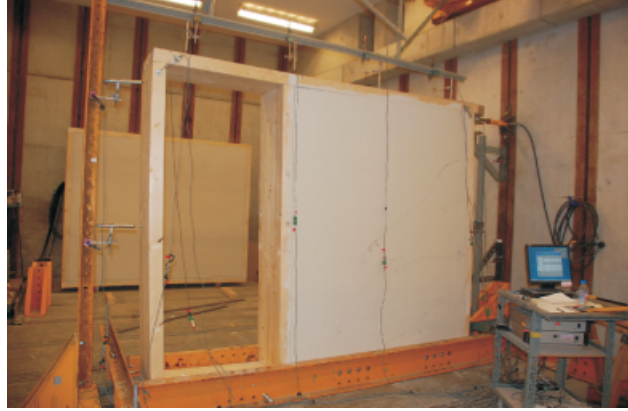


Figure 3. Racking test on a two-bale panel

Under testing, the adopted design serviceability limit was a horizontal timber head plate displacement of $h/500$; for example, 6 mm for the 3 m high panels. Under racking load, a composite action is evident between the timber frame and rendered straw infill. Initially the timber carries all loading, but after 2 – 3 mm lateral displacement the timber frame bears onto the lime render. In the panel system the straw is always recessed inside the timber frame, which provides a guide for render thickness as well as allowing it to bear onto the timber frame during racking. This contact enhances the stiffness of the panel. The render is much stiffer than the straw (measured values for $E_{\text{render}} = 1000 \text{ N/mm}^2$ and $E_{\text{straw}} = 1 \text{ N/mm}^2$) and so is assumed to carry all of this additional loading. However, the straw plays an important secondary role in restraining the render and preventing premature buckling failure of the render which has a slenderness of close to 100 (ratio of render height to thickness). In testing, the three-bale panels were approximately three times stiffer than the two-bale panels (Gross, 2009). Following refinements in design, the three-bale panels now achieve a lateral stiffness of 6.4 kN/mm with lateral load capacity at $h/500$ displacement of 19.2 kN/m. Under repeated laboratory testing it was found that cracking of the render does not occur until displacement exceeds $h/300$,

providing a factor of safety against cracking of at least 1.25 compared to serviceability loading at $h/500$. However, render cracking does not constitute ultimate structural failure as considerable further post-cracking ductility is derived from the composite action between the infill and frame.

4 Numerical modelling of panels

Numerical models of the structural panels have been developed to facilitate the future design and analysis of complete ModCell building structures. Full-scale tests are expensive and time consuming to conduct, so numerical modelling offers the opportunity for exploring future innovation while minimising the need for further extensive physical testing. Two numerical models were developed: a finite-element model (FEM) and a simpler linear spring model. The models specifically allow the racking behaviour of the panels to be analysed, as this is the limiting aspect of structural panel performance. Both models are linear elastic and do not attempt to predict material failure. This is not seen as a limitation from the design perspective since panel racking behaviour is linear up to the deflection serviceability limit. Beyond the serviceability limit, design capacity is defined by deflection limits that control render cracking.

The FEM (Figure 4), created using Robot Millennium software (Gross, 2009), uses known material properties and is validated against test results with good correlation. As the FEM is linear elastic the material models created for it are also linear elastic (Table 1). They are created from known mechanical properties (such as bending, compressive and tensile strengths, elastic and shear moduli) for each of the three materials modelled: glue-laminated timber, lime-based render and stainless steel reinforcing bars. Render skin buckling is not included in the model as the bond between the straw infill and the render has been shown to prevent this very effectively. The FE mesh used to model the render was created using the software's automatic mesh function where the most suitable meshing criteria are selected; in this case Coon's method was used and returns a weighted coefficient of mesh quality of 0.89.

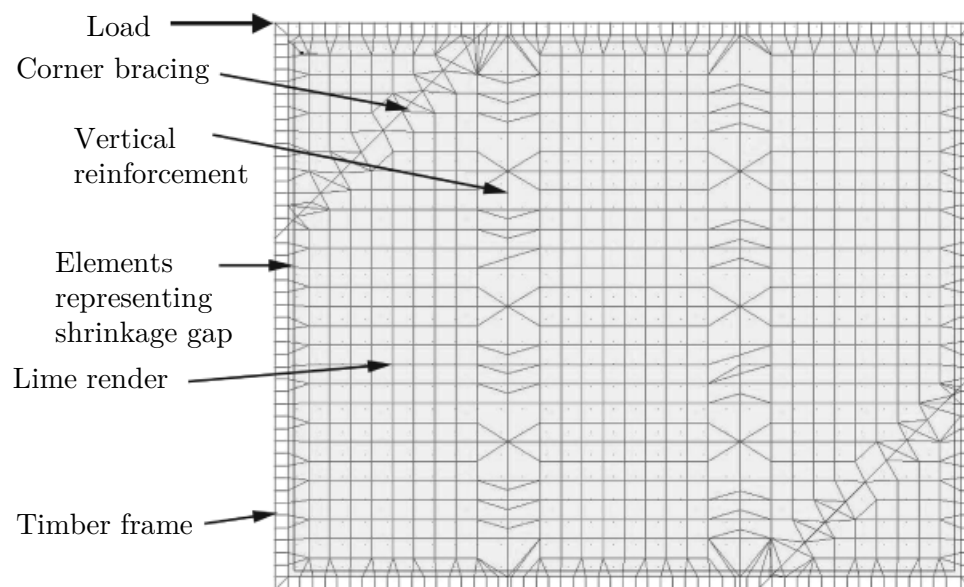


Figure 4. Three-bale FEM

Table 1. FEM element summary

Element description	Element type	Cross-section	Material properties
Timber frame	Lime render	490 mm deep \times 100 mm high	$E_{parallel} = 11\,000\text{ N/mm}^2$ Shear modulus, $G = 690\text{ N/mm}^2$ Bending strength = 24.0 N/mm^2 Axial tension = 14.0 N/mm^2 Transverse tension = 0.5 N/mm^2 Axial compression = 21.0 N/mm^2 Transverse compression = 2.5 N/mm^2 Shear strength = 2.5 N/mm^2
Corner bracing	Lime render	Area equivalent to two 12 mm diameter bar	$E = 200\,000\text{ N/mm}^2$ Poisson ratio, $\nu = 0.3$ Shear modulus, $G = 76\,923\text{ N/mm}^2$ Calculation strength = 200 N/mm^2
Vertical reinforcement	Lime render	Area equivalent to two 10 mm diameter bars	Steel bar – as corner bracing
Lime render	Panel	60 mm total thickness	$E = 5000\text{ N/mm}^2$ Shear modulus, $G = 2000\text{ N/mm}^2$ Calculated compressive strength = 2.0 N/mm^2 (from laboratory testing)
Shrinkage gap	Lime render		$E = 0.01\text{ N/mm}^2$ Poisson ratio, $\nu = 0$ Shear modulus, $G = 0.01\text{ N/mm}^2$ Calculated strength = 500 N/mm^2

The cross-spring model was created using Oasys GSA 8.4.0.17 software (Figure 5) and is an empirical model that equates the measured stiffness of the laboratory test panels to an equivalent spring stiffness. The timber frame was modelled as a series of pinned members with two springs providing racking stiffness. The stiffness values of the springs used to model the corner-braced three-bale panel and the two-bale panel are 6475.7 kN/m and 3387.5kN/m respectively. Unlike the FEM, which allows more complex panel development to be conducted, the cross-spring model was developed primarily as a more practical design tool for the two existing panel types.

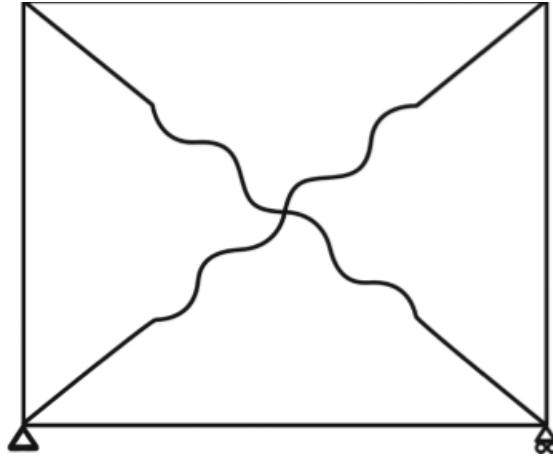


Figure 5. Three-bale cross-spring model

There are notable advantages and limitations to both models, which are summarised below. A significant advantage of the FEM is that it is validated by the panel test results, not empirically based upon them. This allows parameters such as corner joint stiffness to be adjusted and the impact this has on the overall panel stiffness to be understood. A parametric analysis of the panels was completed by Gross (2009) using the FEM described above. It was concluded that, where increased stiffness may be required from a panel, this can be efficiently achieved by increasing the render thickness to 40 mm on each face while also introducing plywood gusset plates in the corners of the frame.

However, an important limitation of the FEM is that it is unidirectional due to the omission of bracing in opposite corners (Figure 4). This means careful consideration of the behaviour of the model is required and different models have to be created to accurately represent the true behaviour of a complete structure under three-dimensional loading. Conversely, the cross-spring model represents the behaviour of the panel irrespective of the loading direction and as such only a single model is necessary. This is a significant advantage since the process of design often requires the panel layout to be altered as the building's form is refined. The cross-spring model accommodates these changes quickly and simply, speeding up the design process. In addition, the FEM requires significantly more computer resources to run each load case, which can further slow this design approach.

Both models were used in the design of the two-storey prototype house described in Section 5 of this paper. For both models an identical wind load was applied to the building model in order to determine the maximum horizontal serviceability deflection at the roof level. The cross-spring model estimated the serviceability deflection under wind load alone to be 4.0 mm, whereas the FEM estimated a maximum of 4.6 mm; both are less than the $h/500$ serviceability criteria of 11 mm. Load testing of the full house was completed to validate these values; however, the actual house is expected to be stiffer owing to the inclusion of an internal solid cross-laminated timber (CLT) shear wall as well as solid timber CLT first floor and roof diaphragms.

5 Wind load testing of a full-scale prototype building

5.1 Test building and arrangement

The BaleHaus (Figure 6) is the first prefabricated straw bale house to use the ModCell panels as load-bearing structural elements. The prototype house was built on the University of Bath campus as part of a two-year research project. The two-storey structure comprises 16 ModCell panels supported on a reinforced concrete floor slab. The first floor and roof are solid 120 mm thick CLT decks supported directly by the straw bale panels. Wall et al. (2012) provide further details on the development, construction and environmental performance of the BaleHaus.



Figure 6. BaleHaus at Bath

On the ground floor, two 100 mm thick internal cross-laminated walls contribute to horizontal bracing and support of the floor plate. Upon installation, the panels are initially fixed together at the top using 8 mm diameter screws driven at 45 degrees from one panel to the next. The sides of the panels are then connected using 200 mm wide and 12 mm thick plywood panels that are screwed to the glulam frames using 60mm long screws at 200 mm centres along the length of the panel. Shear and uplift forces are transferred from the panels to a timber sole plate. At ground floor level this sole plate (Figure 7) is fixed to the concrete slab using mechanical anchor bolts at 600 mm centres. At first floor level 8 mm diameter screws that ensure a 100 mm embedment in the connecting timber are used to screw the sole plate to the CLT floor plate. The CLT floor plate is screwed to the panels below in the same manner prior to installing the sole plate.



Figure 7. Fixing of panel to sole plate

BaleHaus represents the first full structural application of ModCell panels. Methods of connecting adjoining panels together and the overall structural performance of the house had not been previously tested. BaleHaus was designed using the cross-spring model detailed above with the design wind load determined in accordance with BS 6399-2: 1997 (BSI, 2002). Based on an effective wind speed of 37 m/s, an overall characteristic wind load of 35 kN (equivalent to $+0.77 \text{ kN/m}^2$ windward and 0.45 kN/m^2 leeward) was used. As BaleHaus is intended to provide a representation of a generic house, it is important that the design is suitable for applications in as many sites as possible across the UK and elsewhere.

Having a full-scale prototype offered the unique opportunity for lateral load testing of the whole structure to assess performance and validate the structural models used in design. The load testing of the house sought to apply a static lateral load of up to 40 kN, representing just over 1 kN/m^2 loading, onto one elevation of the house while measuring the deflection response of the opposite elevation. Loading was applied directly to the first floor and roof diaphragms using hydraulic jacks. Three different loading configurations were studied: application of 40 kN at the first floor level only; application of 40 kN at the roof level only; and application of 20 kN simultaneously at both first floor and roof levels. Rather than directly replicate a real dynamic wind event, these controlled loading cases provided the opportunity to carefully evaluate the performance of the house and assess different floor levels independently as well as the global response.

For testing, two separate scaffolding frames were constructed by a local sub-contractor: a reaction frame constructed on the east-facing elevation of the house and a displacement frame on the west elevation (Figure 8). The reaction frame was designed on the basis of an applied characteristic load of 40 kN total load at a height of 6m, with the design ultimately determined by the connection capacity of the scaffolding frame.



Figure 8. Test set-up

Loading was controlled using a single hand-operated Enerpac pump that operated four load jacks connected to the scaffolding frame. The four jacks were spaced at 2 m intervals centered on the house, with load bearing onto the solid timber floor and roof and the solid timber spacer around the stair core, which sits on top of the ModCell panels. Only two jacks were used during the separate floor and roof diaphragm tests. Testing was undertaken six months after completion of construction. During testing, no additional variable action (live loading) was applied to the house. The permanent (dead) load of the house (self-weight) is approximately 35 t.

Draw wire transducers, attached at 16 different positions around the BaleHaus, measured the lateral and vertical displacement of the top and bottom panels and were used to determine racking stiffness, global sliding of the structure and separation of the panels at ground and first floor on the windward side. Four transducers were placed at ground level, first floor and roof level on the west elevation and were used to measure horizontal displacement. Two transducers were fixed to the ground at the outside edge of the elevation with the draw wire attached to the house vertically upwards, with a further two transducers placed at the first floor level with the wire attached to the cladding above the floor level. These were used to measure rotation and vertical separation of the panels. Displacement was measured to the nearest 0.1 mm and applied load to the nearest 0.01 kN. In addition to the draw wire transducers the displacement was also measured using two theodolites that were targeted onto the house. Linear

rules were fixed to the east edge of the south elevation, at the first floor and roof level; this allowed measurement of the displacements to be manually taken through the theodolites.

5.2 Results of loading tests

The load–deflection response at roof level of the BaleHaus to the three different load cases is shown in Figure 9. As expected, the worst case loading was 40 kN applied at the roof level; the maximum total horizontal displacement recorded under 40 kN in this test set-up was 3.5 mm, measured at the top corner of the leeward face of the building. Owing to the nature of the test a certain level of noise can be expected from the displacement values recorded. This is evident in Figure 9, which shows horizontal deflection at roof level owing to racking only with the sliding component of the total displacement removed. From inspection of the load–displacement plots it can be seen that in general the displacement response of the building is broadly linear elastic. During different tests, up to 2 mm of horizontal sliding at the base and 1.5 mm of floor separation was recorded by the draw wire transducers.

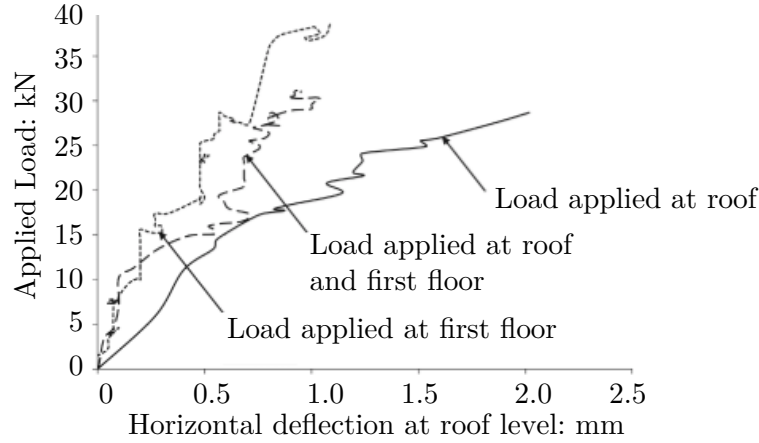


Figure 9. Deflection at roof level

Analysis of the test data indicated that the building was approximately 2.5 times stiffer under static lateral loading than predicted through a spring model analysis of the building. There are several possible sources of the increased stiffness displayed by the complete building. Most importantly, the internal structure of the building included solid CLT first floor and roof diaphragms supported by an internal CLT shear wall and stair well. The external cedar cladding is also likely to contribute some unknown additional

stiffness. Inclusion of these elements into the cross-spring model is discussed below.

5.3 Discussion of loading test results

Following the testing it was possible to validate the original cross-spring model. This was achieved by modifying the original cross-spring model of the BaleHaus. The solid timber floor and roof as well as the internal shear walls that were previously omitted were added, by modelling them using the FEM. The loading regime was then adjusted to represent the four point loads that were applied to the actual house in the three different combinations, as in the testing described above.

The modelling allowed for a comparison between predicted and test results from Figure 9. The comparison showed a strong correlation between the modelled behaviour and actual behaviour (Figure 10), with a correlation coefficient of 0.94, 0.96 and 0.99 for the first floor, top floor and combined loading cases, respectively.

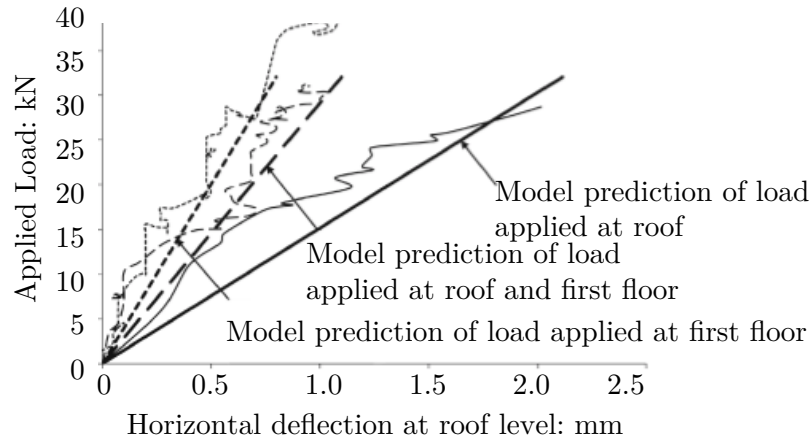


Figure 10. Comparison of roof level deflection model with test results

Under testing, the BaleHaus is sufficiently stiff to withstand a 40 kN simulated wind load, deflecting only 3.5 mm at roof level. The building performs significantly better than its design requirements of an applied wind load of 35 kN and a serviceability limit of 11 mm deflection based on a $h/500$ deflection criterion. This suggests there is scope to modify the original design parameters to further maximise the efficiency of the structural form. However, since the testing showed that the BaleHaus is stiffer than required under the design wind load for the site, there is also scope to use the original design in more severely exposed sites.

It has been demonstrated that the cross-spring model is a reliable model for predicting the actual behaviour. It is suitable for the design of future load-bearing ModCell structures. The cross-spring model estimates that the maximum wind load that can be applied to the house is 100 kN, which is limited by the serviceability requirement of $h/500$, and with the panels still behaving within the elastic range. This wind load is equivalent to a basic mean wind velocity of 45 m/s. This means that the design of the BaleHaus, according to BS EN 1991-4 (BSI, 2005) is widely applicable to locations throughout the UK.

Only one three-bale panel is required on the ground floor in each orthogonal direction to provide sufficient lateral stability in the present site location. There is therefore scope for alternative panel arrangements and a different structural strategy. This could lead to a different architectural style, which differs from the panelised system that is currently used. However, adopting this ideology would sacrifice many of the benefits of using straw bales, principally for their internal environmental control and carbon sequestration, which would be limited if only using two three-bale panels. The internal shear wall could perhaps be completely removed or simply decoupled so that there is no additional shear resistance provided from internal walls. The shear wall is constructed from 100 mm thick three-ply CLT and provides considerable in-plane stiffness. Taking the model used to validate the testing and removing the internal shear wall gives a deflection of 5.3 mm, which is approximately half the serviceability deflection.

The capacity of the panels is sufficient for additional floors, although beyond three floors there is a possible concern of disproportional collapse, in which case a structural frame may need to be adopted. The total wind load to BS EN 1991-4 (BSI, 2005), for a three-storey BaleHaus on the same Bath site, is 41 kN. Taking the model used to validate the testing and adding the same panel arrangement as the first floor to create the second floor and applying the total wind load produces a total deflection of 4.3 mm, significantly less than the serviceability requirement of $h/500$ of 18 mm.

6 Conclusion

The success of straw bale construction in the mainstream sector is in part dependent on confidence with this non-conventional material. Previous research has focused on the improvement of individual panels and has not

considered the way in which these panels work together in a complete structure. A full-scale lateral load test of the BaleHaus was conducted and showed that the structure was stiffer than the make-up of the individual panels would have suggested. Structural computer models were created in aid of the analysis and the design of structures made from the panels. The cross-spring model was validated against the testing of the house. Therefore, future use of the modelling procedure would be suitable for different arrangements of the panels; the cross-spring model has recently supported successful delivery of a series of three-storey ModCell buildings in Leeds (LILAC housing). A parametric study of the design choices has shown the potential of prefabricated straw bale panels to be used structurally within the UK.

The work presented here focused solely on prefabricated straw bale construction; however, the scope of the results is applicable to other forms of timber-based panel construction increasingly adopted for bio-based materials. In particular, there are similarities with hemp-lime and cellulose fibre insulated prefabricated timber panels. To be successful, ModCell must compete with existing market-based solutions including insulated cavity masonry walling and timber stud wall solutions. Although ModCell panels are able to demonstrate comparable or better performance, construction costs and lack of certification remain a significant barrier to wider development. Further development of the ModCell system is therefore now focusing and supporting the process of obtaining CE product certification. The process of certification requires structural fire testing under load and demonstration of long-term durability. Further research and innovation is also exploring alternatives to the make-up and finishes of the panels.

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